State of California
Agency

Business Transportation and Housing

**MEMORANDUM**

To: Gustavo Dallarda
Program Manager

Date: March 30, 2001

File: 11-IMP-111
KP R28.2/R32.5
11-199361

From: **DEPARTMENT OF TRANSPORTATION**
**ENGINEERING SERVICE CENTER –**
**DIVISION OF STRUCTURAL FOUNDATIONS**

Subject: GEOTECHNICAL DESIGN REPORT FOR SR 111, R28.2 / R32.5

In accordance with the request from Project Development S-1, enclosed for your consideration is the Geotechnical Design Report for Phase 3 of State Route 111. The report defines the geotechnical conditions determined by the site investigation program. Additionally, the report presents geotechnical design recommendations including specific construction criteria for the subject project.

The accompanying report for Phase 3 of the project is based on the report for Phase 2 that was originally authored by Martin Skyrman of Roadway Geotechnical Engineering South and issued in March, 2000. The basic format of the Phase 2 report has been retained and changes have been made as appropriate to Marty's report in order to reflect any differences that have been made evident by the results of our Phase 3 investigation. During the investigation for Phase 3 of the project, shallow ground water conditions (less than 1 m below existing grade) were observed at one of the boring locations. This could potentially cause some problems during construction. Recommendations to address this problem are included in Section 8.3 of the report.

# GEOTECHNICAL DESIGN REPORT

**FOR THE**

**CONSTRUCTION OF FOUR-LANE DIVIDED EXPRESSWAY NEAR BRAWLEY FROM 0.1 KILOMETER NORTH OF KEYSTONE ROAD TO 0.5 KILOMETER NORTH OF MEAD ROAD**

**STATE ROUTE 111 (UNIT 3)**

**11-IMP-111**
**KP R28.2/R32.5**
**11-199361**


**California Department of Transportation**
**Office of Materials and Foundations**
**Roadway Geotechnical Engineering**
**San Diego, California**

**March 2001**

If you have any questions regarding this report , please call the undersigned at (858)467-4054.

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**11-IMP-111**
**KP R28.2/R32.5**
**11-199361**

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## 1. Introduction

The proposed project consists of upgrading State Route 111 (SR-111) in Imperial County from a two-lane conventional highway to a four-lane divided expressway between Ross Road, near the City of El Centro, and the easterly intersection with State Route 78 (SR-78), in the City of Brawley (see Figure 1 – Vicinity Map). This report pertains to Unit 3 of the proposed project, which extends from 0.1 kilometer north of Keystone Road (KP 28.2) to 0.5 kilometer north of Mead Road (KP 32.5). The project consists of three units and will enhance capacity and relieve truck traffic on State Route 86 (SR-86) through El Centro (see Figure 2 – Project Map).

Unit 3 consists of the construction of northbound and southbound roadbeds (two lanes each) and rehabilitation of existing SR-111 for use as a frontage road. Also included in the project are the improvement to Schartz Road and construction of additional frontage roads for property access near Schartz Road and Mansfield Road and cul-de-sacs at Carey Road and Mead Road. The project also includes a new bridge over Rockwood Canal. The investigation phase of the Geotechnical Design Report (GDR) was comprised of review of published data, site reconnaissance, drilling exploratory borings, and in-situ testing.

The purpose of this report is to document subsurface geotechnical conditions, provide analyses of anticipated site conditions as they pertain to the project described herein, and to recommend design and construction criteria for the roadway portions of the project. This report also establishes a geotechnical baseline to be used in assessing the existence and scope of any changed conditions that may be claimed.

This report is intended for use by the project roadway design group, construction personnel, bidders and contractors.

## 2. Existing Facilities and Proposed Improvements

The proposed alignment is planned as an embankment section along its entire length with the road surface elevation a maximum distance of approximately 2.0 m above existing ground, including the structural section. Per the preliminary design plans for the project, the planned structural section for the main lanes of State Route 111 is approximately 1.2 m thick. 600 mm of subexcavation and recompaction is planned for the main lanes to improve strength of the subgrade soils.

Existing State Route 111 is a two-lane conventional highway with 3.7 m wide lanes and 1.2 m wide paved shoulders without dikes. State Route 111 was added to the State Highway System in 1933 and was maintained as a macadamized road until 1948 when the initial roadway of asphalt concrete pavement was constructed. State Route 111 was added to the Freeway and Expressway System in 1959. Existing land use along the proposed project corridor is primarily agricultural lands that are irrigated and drained using a network of subsurface tile drains and a series of open concrete lined and unlined canals and drains operated by the Imperial Irrigation District (I.I.D.). These facilities (operated by I.I.D.) cross under the existing highway at many locations by means of culverts or siphons with associated headwall, endwall, or ditch excavations as close as 1.5 m to the existing traveled way.

Along much of the highway there are adjacent overhead powerlines, and overhead and underground communication facilities. The Mesquite lake Power Plant is located 0.4 km south of Keystone Road to the west of existing SR-111.

There is a history of drainage related problems in this length of State Route 111. Subsurface drainage is particularly troublesome where irrigation canals lie adjacent to the roadway with water levels rising above the road grade. Irrigation water seeps through gaps and cracks in the concrete canal lining and into the subgrade and structural section of the road. The saturated soil loses strength and pumps when traffic loads are applied, which initiates pavement cracking and further degradation of the roadbed. Clearly, a raised roadbed with a finish grade above the water levels in adjacent ditches, is necessary to avoid moisture related distress to the structural section.

Another related problem stems from the proximity of deep drain ditches to the roadway. Subsurface flow beneath the roadway causes slumping or piping of the sediments on the ditch slope face, eventually undermining the road. There also exist localized conditions of ponding and locations near the road shoulder that remain consistently wet due to year-round irrigation. Positive drainage away from the roadway is lacking, resulting in degradation of the existing structural section.

The Imperial Fault, which crosses State Route 111 about 8.9 kilometers south of Keystone Road, and the nearby Brawley Seismic Zone, located within a few kilometers of the project, have aggravated roadway surface conditions and drainage in the past. Several nearby crossroads dropped nearly 0.3 m in elevation during the last major earthquake in the area, which occurred in October 1979. Pavement cracking across the road at the intersection with the Imperial Fault is an ongoing maintenance problem due to continuous fault creep.

The main lanes for State Route 111 are planned to be made up of 7.2 m traveled way width in both directions with 1.5 m inside median shoulders and 3.0 m outside shoulders. Outside side slopes will have slope ratios of 1:6 or flatter. The main lanes are based on the "G" baseline, which extends approximately from Station 295+00 (0.1 km north of Keystone Road) to Station 342+00 (0.5 km north of Mead Road). Schartz Road improvement extend from Station 9+00 to 13+00. Proposed frontage roads, cul-de-sacs and sound berms consist of the following:

- Frontage road at intersection of existing SR-111 with Schartz Road (approximately from Station 10+00 to Station 17+32 "G" line)
- Frontage road north of Schartz Road and east of the proposed main alignment (approximately from Station 10+00 to Station 22+00 "G" line)
- Cul-de-sacs on Carey and Mead Roads

- Sound berm (rising approximately 1.75 m above roadway elevation) along the northbound lanes of the proposed main alignment ("G" line from Station 332+00 to Station 334+00)
- A new bridge over Rockwood Canal

## 3. Pertinent Reports and Investigations

1. Materials Design Report, 11-IMP-111, KP R28.2/R32.5, *in progress*, by California Department of Transportation, District 11, Pavement Engineering Section.

2. Project Report, September 13, 1994, by Rick Engineering Company for California Department of Transportation.

3. Preliminary Engineering Geology and Geologic Hazards Report, July 1992, by California Department of Transportation District 11 Materials Laboratory.

4. Preliminary Materials Information for Project Study Report (PSR), October 22, 1989, by California Department of Transportation, 11-IMP-111, PM 8.2/22.0, 11-910054, 5958 111 St.

# 4. Physical Setting

## 4.1 Climate

The Imperial Valley climate is extremely arid. The mean annual rainfall at the Imperial weather station, located approximately 14.5 km from the project, is 7.6 cm. The mean annual temperature is 22.8°C. During the months of May through October, the average maximum daytime temperature exceeds 37.8°C.

## 4.2 Topography and Drainage

The Imperial Valley is a broad, flat lying expanse with little topographic relief and contains the Salton Sea, a base level sink that is 76.2 m below sea level. The region's natural ground surface was formed as a lake floor and is therefore relatively flat with a gentle northward dip towards the Salton Sea. Within the project, the low ground is at the south end near Keystone Road where the elevation is approximately 43 m below sea level. The topography slopes gently northward and the elevation at the north end of Unit 3 (Mead Road) is approximately 37 m below sea level.

Surface runoff along the project alignment is by sheet flow and empties into a network of irrigation drain ditches, which in turn empty into the Alamo River and eventually into the Salton Sea. Drainage is orderly due to the construction of subsurface drains (tiles) and drain ditches in the surrounding farmfields. Subsurface tiles are located approximately 1.5 m below existing ground surface. A regional perched ground water table exists at approximately the depth of the tile drains.

Farmfields are irrigated such that a positive pore pressure regime is created with a downward gradient toward the tile drains. This practice prevents the accumulation of salts, which has a detrimental impact on agriculture. Ponding exists locally, particularly along the roadway and usually on the opposite side from a drain ditch.

### 4.3 Man-made and Natural Features of Engineering and Construction Significance

Agricultural water supply canals and farm field drains cross the area. Tile drains criss-cross the farm fields about 1.5 m below the ground surface. Locations of field drains are on file at the Imperial Irrigation District office.

An underground gas line operated by Southern California Gas Company runs parallel to the proposed alignment and within the right-of-way from 1.0 km south of Keystone Road to Keystone Road.

### 4.4 Regional Geology and Seismicity

The project corridor lies in the Salton Trough geomorphic province. This is one of the 11 recognized geomorphic provinces of California. The Salton Trough is an incipient rift valley and is an inland extension of the Gulf of California. The Colorado River blocked the inland extension of the Gulf of California with deep deposits of delta sediments. The Salton Trough is local base level and is below sea level.

The Salton Trough is an active spreading center, a tectonic basin formed by faulting and the rifting of tectonic plates. Two separate basins, the Salton Sea and the Mesquite Basin, have formed as a result of two continental-size, tectonic plates slowly drifting apart. A series of right-lateral en echelon faults are present due to the rifting process. The overall trend of these faults is northwesterly (see Figure 4.4.1 – Regional Fault Map).

Locally, Colorado River sediments were deposited within an inland lake. A former shoreline of one such antecedent lake, known as Lake Coahuilla, is evident, at about sea level, outside of the project limits. Lake Coahuilla occupied this trough periodically for thousands of years depositing at least 30 m of Quaternary lake bed sediments. These lake sediments are unconsolidated and comprised mostly of clay, silt and fine to very fine-grained sand.

Within the subject area, most of the surface sediments have been disturbed or reworked by extensive farming operations. Most near surface sediments along the proposed roadway are loose, deeply tilled soils that can be rich in organic content. Fill material, used to elevate land for either irrigation channels or for farm fields, is also present.

Though the Imperial Fault does not cross the proposed Unit 3 alignment (it crosses 4 km to the south), it warrants discussion due to its proximity to the site. Between 1900 and 1940, the Imperial Fault is believed to have had a damaging earthquake once every 11 years. The last two major earthquakes on this fault in May 1940 and in October 1979, had almost identical ground surface ruptures.

Four other major faults lie within approximately 18 km of the project limits: 1) Brawley Fault Zone; 2) the Superstition Hills Fault; and 3) the Superstition Mountain Fault. In addition, the Elsinore fault lies approximately 46 km to the west of the project. All are considered active (movement within the past 11,000 years).

The principal transform fault between the tectonic plates is the San Andreas Fault. It is traceable on ground surface as far south as Bombay Beach, about 48 km north of the project. However, some believe that the Brawley Seismic Zone, which lies within a few km of the project, may express the subsurface extension of the San Andreas Fault. The zone is characterized by swarms of small earthquakes, which rumble repeatedly for weeks at a time. The Brawley Seismic Zone appears to connect the San Andreas Fault with the north end of the Imperial Fault.

Table 4.4.1 summarizes names of known active faults, distances from the project site, and maximum credible earthquake magnitudes.

**Table 4.4.1 – Deterministic Site Parameters** (from Thomas Blake's EQFAULT Software 1996 and from CDMG Open file report (OFR)92-1)

| Fault Name | Approximate Distance (km) | Maximum Credible Magnitude |
|---|---|---|
| Borrego Mountain (San Jacinto) | 46 | 6.5 |
| Casa Loma-Clark (San Jacinto) | 71 | 7.0 |
| Coyote Creek (San Jacinto) | 75 | 7.0 |
| Elsinore | 46 | 7.5 |
| Hot S-Buck Rdg. (San Jacinto) | 93 | 7.0 |
| Imperial - Brawley | 4 | 7.0 |
| San Andreas (Coachella Valley) | 54 | 8.0 |
| Sand Hills | 30 | 8.0 |
| Superstition Hills (San Jacinto) | 9 | 7.0 |
| Superstition Mtn. (San Jacinto) | 13 | 7.0 |

The Imperial Fault is believed to be capable of generating a Maximum Credible seismic event with a magnitude of 7.0. Other faults within a 100 km radius, such as the Superstition Hills and Superstition Mountain Faults, may also generate strong ground shaking within the project.

## 4.5 Soil Survey Mapping

The Soil Survey of Imperial County California, Imperial Valley Area by the U.S.D.A Soil Conservation Service, 1981 was used to map soils encountered along the project corridor. Shown on Table 4.5.1 are the soil units that are encountered along the project corridor along with a summary of their engineering properties. The Soil Survey only maps surface soils to a depth of 1.5 m.

In addition to those values presented in Table 4.5.1, all soil types exhibit a high risk of corrosion to uncoated steel and a moderate to high risk of corrosion to concrete.

## Table 4.5.1 – Soil Parameters from SCS Soil Survey Mapping

| Soil Map Symbol | Soil Name | Depth (meters) | USCS Classification | Percent Fines (<No. 200 sieve) | Liquid Limit | Plasticity Index | Soil pH | Depth to Water (meters) | Shrink-Swell Potential |
|---|---|---|---|---|---|---|---|---|---|
| 110 | Holtville silty clay, wet | 0-0.4 | CH-CL | 85-95 | 40 - 65 | 25-35 | 7.4 -8.4 | 0.9-1.5 (perched) | High |
| | | 0.4 0.6 | CH - CL | 85-95 | 40 – 65 | 25-35 | 7.4 -8.4 | | High |
| | | 0.6 – 0.9 | ML | 55 – 85 | 25 – 35 | NP – 10 | 7.4 -8.4 | | |
| | | 0.9-1.5 | SM, ML | 20 – 55 | - | NP | 7.4 -8.4 | | |
| 114 | Imperial silty clay, wet | 0-0.3 | CH | 85-95 | 50-70 | 25-45 | 7.9-8.4 | 0.9-1.5 (perched) | High |
| | | 0.3-1.5 | CH | 85-95 | 50-70 | 25-45 | 7.9-8.4 | | High |
| 115 | Imperial-Glenbar silty clay loams | 0-0.3 | CL | 85-95 | 40-50 | 10-20 | 7.9-8.4 | 0.9-1.5 (perched) | High |
| | | 0.3-1.5 | CH | 85-95 | 50-70 | 25-45 | 7.9-8.4 | | High |
| 122 | Meloland very fine sandy loam | 0-0.3 | ML | 55-85 | 25-35 | NP-10 | 7.4-8.4 | 0.6-0.9 (perched) | Low |
| | | 0.3-0.7 | ML | 50-70 | 25-35 | NP-10 | 7.4-8.4 | | Low |
| | | 0.7-1.8 | CH,CL | 85-95 | 40-65 | 20-40 | 7.4-8.4 | | High |

# 5. Exploration

A geotechnical investigation was conducted to determine material types and characteristics within the proposed project corridor. The investigation was comprised of drilling with Standard Penetration Testing (SPT), and Cone Penetrometer Testing (CPT).

## 5.1 Drilling and Sampling

Eleven small-diameter borings were drilled in November and December 2000 to help define material types, depths, ground water levels and engineering properties of site soils. The borings were performed using mud rotary drilling to a depth of 9.6 m. Prior to drilling, the Geotechnical Engineering Section determined that a boring depth of approximately 9.6 m was sufficient to characterize soil properties and geotechnical conditions along the project corridor. Drilling was performed with a State furnished Mobile B-47 drill rig. Boring locations are identified on Figure A.1 in the Appendix and within the header region of the Logs of Test Borings located in the Appendix.

Disturbed samples were obtained by Standard Penetration Testing (SPT). Undisturbed samples were taken by a modified California Sampler. The Modified California Sampler is comprised of a 63.5 mm diameter barrel that holds six 152.4 mm brass sleeves. Sampling intervals varied with depth slightly between borings, however, intervals for both SPT and undisturbed sampling was generally 1.5 m. SPT blow counts were used to determine soil consistency or relative density. Samples were trimmed flush with the sleeve's ends; these ends were covered with plastic caps and the resulting seams were taped. All samples are currently stored at the District 11 Materials Laboratory.

Nine of the eleven borings were developed as observation wells to characterize ground water conditions.

### 5.2 Instrumentation

A total of eleven observation wells (standpipe piezometers) were installed during the drilling program to define ground water conditions along the project corridor. The observation wells are comprised of 50 mm diameter slotted PVC pipe backfilled with sand throughout their entire depth with a bentonite plug at the ground surface. In addition to the nine wells developed at boring locations, another two observations wells (P1 and P2) were installed at the locations shown in Figure A.1.

### 5.3 Exploration Notes

No adverse conditions were encountered during our subsurface investigation. There were no impediments to drilling and/or sampling.

## 6. Geotechnical Testing

### 6.1 In Situ Testing

In situ testing consisted of Standard Penetration Tests (SPT). SPT blow counts, shown on the Logs of Test Borings in the Appendix, were used to determine consistency or relative density. Standard Penetration Testing conformed to standard test method ASTM D-1586. The descriptors of soil consistency/relative density derived from SPT blow counts conform to the criteria presented in the California Department of Transportation's Soil and Rock Logging Classification Manual (1996). Soil classifications conform to the Unified Soil Classification System.

### 6.2 Laboratory Testing

No laboratory testing was performed on samples obtained during the investigation program since adequate test data on representative soil samples had been developed in Phase 2 of the project. Soil conditions were not deemed to be significantly different to warrant additional testing.

# 7. Geotechnical Conditions

## 7.1 Site Geology

During our field reconnaissance and exploratory subsurface evaluation, Quaternary (DMG, 1962) lacustrine sediments were found to exist along the entire length of the project corridor. This is in agreement with the geologic mapping portrayed in Figure 7.1. The lacustrine (lake bed) deposits were derived from the historical Lake Cahuilla, a lake created by the influx of Colorado River water into the Salton Trough Basin. These deposits were encountered to the maximum explored depth in all eleven exploratory borings. These materials are generally comprised of poorly graded fine sand, silty sand, sandy silt, silt, lean clay, and fat clay.

### 7.1.1 Structure

The project site is located in the Salton Trough, a tectonically active, incipient rift valley. The valley is criss-crossed by numerous northwest trending right-lateral strike-slip faults and westerly trending normal faults. The active Imperial Fault crosses existing SR-111 south of the Unit 3 alignment at KP 19.2. This fault trace trends northwesterly and diverges from SR-111 with increasing kiloposts. Three other major active faults lie within 7 km of the project limits – the Brawley Fault Zone, the Superstition Hills Fault, and the Superstition Mountain Fault.

The project site is underlain by Quaternary (DMG, 1962) lacustrine (lake bed) deposits. These deposits are comprised of sands, silts and clays. Bedding in these relatively young geologic materials is predominately flat-lying or approximately so, except in the vicinity of active faulting, where the bedding may be structurally deformed.

## 7.2 Subsurface Soil Conditions

Lake bed deposits within the project area consist of fine sands, silts, lean clays and fat clays. From a roadway design and construction perspective, there is no significant variation in the soil profile from one boring location to the next. On an average, the upper 6 to 8 meters generally consist of sand and silt with few exceptions. Clays are generally present at some depth and their consistencies increase from firm to very stiff. Though some quantity of fat clays will likely be present in the roadbed embankment and subgrade, the variations in moisture content that frequently lead to roadway heave and its associated pavement distress are not expected to occur due to the area's constant hydrologic regime.

Sands are typically fine-grained, poorly graded, medium dense, moist to wet, and mixed with a considerable amount of fines (silts/clays). The standard penetration testing indicate that silt and sand layers are relatively denser at depth, yet there is still potential for localized liquefaction due to saturated conditions and the high degree of seismicity in this area.

The majority of the near-surface materials have been reworked by farming operations. As a result, the top 1.5 m of existing ground will require densification to provide a suitable roadbed foundation. The Materials Report (prepared by the District 11 Materials Laboratory) for this project presents the recommended foundation treatment(s). At the time of this writing, it appears that the foundation treatment(s) will be the same as that presented for Unit 1 and 2 construction. A summary of the recommended foundation treatments(s) for Unit 1 and

2 is contained on page 27 of this report. Organic content in these surface soils does not appear excessive and thus, over-excavation and disposal of these materials is not warranted.

No drilling difficulties were encountered during subsurface investigation. At the time this report is being written, we are not aware of any contaminated soils present along the project alignment.

## 7.3 Water

### 7.3.1 Surface Water

Surface runoff along the project alignment is by sheet flow and empties into a network of irrigation drain ditches, which in turn empty into the Alamo River and eventually into the Salton Sea. Drainage is orderly due to the construction of subsurface drains (tiles) and drain ditches in the surrounding farmfields.

### 7.3.2 Ground Water

Ground water was encountered in all thirteen of the observation wells that were installed during drilling operations. Based on our knowledge of the area and the hydrologic environment imposed by farming operations along the project alignment, the ground water that we encountered represents a perched water condition. A perched water table can occur when there is a layer with a very low permeability separating the perched, unconfined water from lower unsaturated zones and the main aquifer.

Based on ground water depths encountered, ground water is not expected to significantly affect the proposed roadway construction except at the location of Observation Well P-1 where the depth to ground water was measured at 0.8 m. Table 7.3.2 lists the borings in which monitoring wells were installed, the surface elevation of the borings, and the approximate perched ground water elevations. Measured ground water levels along the project alignment ranged between 0.8 and 3.3 m below existing ground surface. Proposed construction should only disturb the upper 1 m of site soils. Thus ground water should be anticipated at the approximate location of Observation Well P-1. Recommendations to address this problem are presented in Section 8.3.

**Table 7.3.2**

**Ground Water Conditions**

***Ground water elevation observed while developing boring – no piezometer set at this location.**

| Boring No. | Approximate Location along "G" line | Approx. Ground Surface Elevation[1] | Perched Ground Water Elevation (measured 2-24-01)[1] |
|---|---|---|---|
| WP-52* | 311+11, 55 m LT | 60.8 | 59.3 1.5 |
| WP-46 | 313 + 89, 17 m RT | 63.9 | 62.6 1.3 |
| B0012-6 | 296 + 31, 10 m RT | 57.5 | 55.7 1.8 |
| HF0012-1 | 299 + 01, 58 m LT | 57.3 | 56.1 1.2 |
| B0012-7 | 320 + 77, 17 m RT | 61.7 | 59.4 2.3 |
| B0012-8 | 305 + 90, 8 m RT | 59.4 | 56.1 3.3 |
| B0012-9 | 324 + 69, 52 m LT | 61.6 | 59.7 1.9 |
| B0012-10 | 338 + 57, 36 m LT | 63.4 | 60.8 2.6 |
| B0012-11 | 330 + 01, 22 m RT | 62.8 | 60.1 2.7 |
| B0012-12 | 341+62, 1 m LT | 64.3 | 61.7 2.6 |
| P-1 | 302 + 70, 48 m LT | 57.9 | 57.1 .8 |
| P-2 | 307 +57, 59 m LT | 59.8 | 57.5 2.3 |

[1] Elevations are based on NAVD88 Datum (plus 100 m)

## 7.4 Project Seismicity

An EQFAULT (software by Thomas Blake) analysis of the project corridor lists distance from the point of interest to the surface trace of active faults, Maximum Credible Events (MCE), and repeatable horizontal ground accelerations (RHGA). This analysis uses the Campbell and Bozorgnia (1994) ground motion attenuation relationship for a soft site to calculate peak site accelerations. Values of acceleration represent mean plus one standard deviation. The results of this analysis are presented in Table 7.4.1. These results are based on a search of faults within a 100 km radius of the project site.

Map Sheet 45 prepared by Mualchin and Jones for California Department of Transportation, Division of Structures titled "Peak Accelerations from Maximum Credible Earthquakes in California" shows the project area in a zone that could experience peak ground accelerations of approximately 0.7g due to a Maximum Credible Earthquake. Empirical observation from the regions strong motion records reveal that the area has sustained horizontal ground accelerations in excess of 0.85g (1979 M6.4). Map Sheet 45 assigns the Imperial Fault a maximum credible earthquake moment magnitude of 7.0.

The Imperial Fault extends from approximately 10 km south of the U.S.-Mexican Border in a northwest trend for a total length of approximately 69 km.

### 7.4.1 Ground Rupture

Surface rupturing is the offset or tearing of the ground surface by relative displacements across a fault during an earthquake. The Imperial Fault crosses SR-111 south of Unit 3 at Worthington Rd. at approximately KP 19.2 (Station 200+00) as shown on Figure 7.4.1.1. The traces of the Imperial fault cut Holocene alluvium and deposits of Lake Cahuilla (Sharp, 1982). This active fault is a right-lateral strike-slip fault. Historically, the Imperial Fault has seen several instances of surface rupture. The last two large surface ruptures occurred on October 15, 1979 (moment magnitude 6.4) and on May 18, 1940 (moment magnitude 6.9). Relatively minor displacements along the fault have occurred in 1966, 1968, 1971, and 1977.

A 30.5 km segment of the Imperial Fault broke at the ground surface during the 1979 earthquake. In response to the 1979 event, approximately 45 cm of cumulative horizontal displacement occurred during a period of 160 days following the event. It has been estimated that approximately 16 cm of vertical slip occurred during the same period (Sharp et. al, 1982). It is also reported that approximately 3.7 m of lateral offset along the fault occurred southeast of El Centro in response to the 1940 event with greater displacements observed farther to the south (DMG, 1962).

The time interval between major surface ruptures varies with the earthquake magnitude being considered. Earthquakes as large as or larger than the1979 event (M6.4) may occur every 30 to 40 years. Earthquakes the size of that of the 1940 event (M6.9) may occur only once every 700 years or so (SCEC, 1999). The potential for future surface rupture at the site is high.

Figure 7.4.1.2 is a map showing sites of liquefaction, ground failure, and other secondary ground effects caused by the 1979 Imperial Valley Earthquake with relation to the project site (Youd and Wieczorek, 1982). The majority of these secondary ground effects occurred along the stream channels of the New River and the Alamo River where there are loose deposits of sandy soils. None of these ground effects were observed along the project alignment, where the surface soils are generally comprised of stiff clays.

### 7.4.2 Shaking

The intensity of ground shaking is dependent on the magnitude of the earthquake, the distance of the earthquake from the area of interest, and the local soil conditions. The magnitude of seismic acceleration is related to several factors including type of fault, fault rupture length, geologic conditions (soft or hard rock sites), site topography and distance to the causative fault. As an example, a small seismic event close to a site can generate the same acceleration as a larger event that is distant to the site. Site geology for this project is characterized by relatively soft lake bed sediments, which are susceptible to seismic wave amplification.

The maximum recorded acceleration for the M=6.9 El Centro 1940 earthquake was 0.33g (Housner, 1970). Strong-motion records have shown that the 1979 Imperial Valley earthquake produced measured vertical and horizontal surface accelerations as great as 1.74 and 0.81g, respectively (Porcella et al, 1982). One strong-motion station is located at Imperial Valley College (intersection of SR 111 and Aten Rd.) approximately 27 km from the 1979 earthquake epicenter (Porcella et al, 1982). Records here showed a maximum horizontal ground acceleration of 0.63g in response to the 1979 earthquake event (M6.4). Two other nearby stations, also 27 km from the epicenter and adjacent to the project alignment recorded maximum horizontal ground accelerations of 0.85 and 0.81g, respectively. In summary, strong-motion records for the 1979 earthquake show that maximum horizontal ground accelerations greater than 0.5g were measured at 7 stations (out of 43 strong-motion stations within an epicentral distance of 150 km in operation during the main shock (Porcella et al, 1982).

Duration of seismic shaking is related to the length of fault rupture (therefore magnitude), geologic conditions (soft or hard site), distance to the causative fault, and basin and boundary characteristics. A general estimation of the duration of strong ground shaking can be made based on earthquake magnitude and stress history during the event. Seed et al. (1969) estimate this duration to be approximately 25 to 30 seconds for a magnitude 7 (Richter) earthquake. Housner (1970) estimates this duration to be approximately 24 seconds for a magnitude 7 event. In addition to the period of strong ground shaking, a period of lessened shaking follows for a relatively long time (Housner, 1970). Records at several monitoring stations confirm durations of significant shaking (ground accelerations exceeding 0.1g) ranging from 5 to 13 seconds for the 1979 Imperial Valley earthquake (Youd and Wieczorek, 1982).

Recurrence intervals based upon Thomas Blake's EQSEARCH software are presented in Table 7.4.2.

Estimated peak horizontal accelerations for the project site, based on strong motion records for the 1940 and 1979 Imperial Valley earthquakes and Campbell's (1991) attenuation relationship for deep soil and soft rock, were calculated using the EQSEARCH software program. Values are presented in Table 7.4.3 for the purpose of comparison to the peak site acceleration derived from the deterministic analysis of the maximum credible event on the Imperial Fault (Table 7.4.1) and actual (recorded) values at strong-motion monitoring stations.

## Table 7.4.1
## Deterministic Site Parameters (from Thomas Blake's EQFAULT Software 1992 and from CDMG Open file report (OFR) 92-1

| FAULT NAME | DISTANCE (km) | MAXIMUM CREDIBLE MAG. | PEAK HORIZONTAL ACCELERATION | SITE MODIFIED MERCALLI INTENSITY |
|---|---|---|---|---|
| IMPERIAL-BRAWLEY | 4 | 7.0 | 0.84 | XI |
| SUPERSTITION HILLS (SAN JACINTO FAULT ZONE | 9 | 7.0 | 0.60 | X |
| SUPERSTITION MOUNTAIN (SAN JACINTO FAULT ZONE) | 13 | 7.0 | 0.49 | X |
| SAND HILLS | 30 | 8.0 | 0.40 | X |
| ELSINORE | 46 | 7.5 | 0.19 | VIII |
| BORREGO MOUNTAIN | 45 | 6.5 | 0.09 | VII |
| SAN ANDREAS (COACHELLA VALLEY) | 54 | 8.0 | 0.22 | IX |
| CASA LOMA-CLARK (SAN JACINTO FAULT ZONE) | 68 | 7.0 | 0.08 | VII |
| COYOTE CREEK (SAN JACINTO FAULT ZONE) | 72 | 7.0 | 0.07 | VI |
| HOT S-BUCK RDG. (SAN JACINTO) | 89 | 7.0 | 0.05 | VI |

## Table 7.4.2
## Recurence Intervals and Probability of Exceedance for Acceleration/Magnitude (output from Thomas Blake's EQSEARCH software)

| Acc. (g) | No. of Times Exceeded | Ave. Occurrence (#/yr) | Recurrence Interval (years) | Probability of Exceedance | | | |
|---|---|---|---|---|---|---|---|
| | | | | 1 yr | 10 yr | 50 yr | 100 yr |
| 0.01 | 347 | 1.73 | 0.58 | 0.82 | 1.00 | | |
| 0.05 | 83 | 0.41 | 2.42 | 0.34 | 0.98 | 1.00 | |
| 0.10 | 13 | 0.07 | 15.46 | 0.06 | 0.48 | 0.96 | 1.00 |
| 0.15 | 3 | 0.02 | 67.00 | 0.01 | 0.13 | 0.53 | 0.78 |
| 0.20 | 3 | 0.02 | 67.00 | 0.01 | 0.14 | 0.53 | 0.78 |
| 0.25 | 2 | 0.01 | 100.50 | 0.01 | 0.09 | 0.39 | 0.63 |
| 0.29 | 2 | 0.01 | 100.50 | 0.01 | 0.09 | 0.39 | 0.63 |
| **Magnitude** | | | | | | | |
| 4.0 | 628 | 3.12 | 0.32 | 0.96 | 1.00 | | |
| 4.5 | 224 | 1.11 | 0.90 | 0.67 | 1.00 | | |
| 5.0 | 76 | 0.38 | 2.65 | 0.31 | 0.98 | 1.00 | |
| 5.5 | 38 | 0.19 | 5.29 | 0.17 | 0.85 | 1.00 | |
| 6.0 | 17 | 0.09 | 11.82 | 0.08 | 0.57 | 0.99 | 1.00 |
| 6.5 | 6 | 0.03 | 33.50 | 0.03 | 0.26 | 0.78 | 0.95 |

## Table 7.4.3
## Estimates of Site Acceleration based on Strong Motion Records of the 1940 and 1979 Imperial Valley Earthquakes (output from Thomas Blake's EQSEARCH software)

| Event | Depth (km) | Moment Mag. | Site Acc.(g) | Site Modified Mercalli Intensity | Approx. Distance from Site (km) |
|---|---|---|---|---|---|
| 5-19-1940 earthquake | 3.0 | 6.7 | 0.219 | IX | 16 |
| 10-15-1979 earthquake | 3.0 | 6.6 | 0.142 | VIII | 34 |

# 8. Geotechnical Analysis and Design

## 8.1 Dynamic Analysis

The EQFAULT software program by Thomas Blake, 1992, was used to calculate maximum credible earthquakes and horizontal ground accelerations for each fault within a 100 km radius of the project site. The ground accelerations for the maximum credible earthquake were developed using the ground motion attenuation relations of Campbell and Bozorgnia, 1994, for a soft site.

The Imperial Fault is the design fault for this site since it crosses existing SR-111 approximately 2 km south of Worthington Rd. The Maximum Credible Earthquake for this fault is estimated to be a 7.0 magnitude event. The repeatable horizontal ground acceleration (RHGA) associated with this event is 0.59g (see Table 7.4.1).

This ground acceleration of 0.59g was considered in our evaluations of liquefaction potential and earthquake induced settlements.

### 8.1.1 Liquefaction and Seismically Induced Settlement

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Liquefaction occurs in saturated soils, that is, soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles that influences how tightly the particles themselves are pressed together. Prior to an earthquake, the water pressure is relatively low. However, earthquake shaking can cause loosely packed groups of particles to collapse. These collapses, in turn, increase the pore-water pressure between the grains if drainage cannot occur. Liquefaction occurs when the pore-water pressure rises to a level approaching the weight of the overlying soil, whereby the granular layer temporarily behaves as a viscous liquid rather than a solid.

The types of sediments most susceptible to liquefaction are clay-free deposits of sand and silts. Generally, the younger and looser the sediment, and the higher the water table, the more susceptible the soil is to liquefaction. Liquefaction is restricted to certain geologic and hydrologic environments, primarily recently deposited sands and silts in areas with high ground water levels. Liquefaction has occurred most frequently when the ground water lies within 10 m of the ground surface. Liquefaction by itself may not be particularly damaging or hazardous. Liquefaction is only destructive when some form of ground displacement or ground failure accompanies it. For engineering purposes, it is not the occurrence of liquefaction that is of prime importance, but its severity or its capability to cause damage.

The evaluation of the liquefaction potential of the subsurface soils is based on the consideration of the anticipated ground water levels, soil types, soil gradations, relative soil densities, intensity of ground shaking, and duration of shaking. In general, liquefaction potential is greatest for loose, fine sands and decreases with increasing grain size and clay and gravel content.

Our subsurface evaluation indicates that the project corridor is underlain by recent lake bed (lacustrine) deposits consisting of fine sand, silt, and clay. Perched ground water levels are near the ground surface (2 m +/-). Most of these deposits are saturated or nearly saturated, which make them more susceptible to liquefaction. Standard Penetration Testing (SPT) blow count test values were used to assess the relative density of in-place cohesionless deposits. Potentially liquefiable zones of sands and silts were encountered at depths between 2.5 and 9.5 m below the existing ground surface. These soils typically exist in a medium dense to dense state. A strong earthquake event could cause local liquefaction of these layers and relatively minor distortion of the roadbed embankment. A more detailed liquefaction evaluation is unwarranted for this project as there are no structures founded in these layers.

Extreme liquefaction mitigation measures such as deep dynamic compaction and vibro-compaction are probably unwarranted in light of the high costs associated with such mitigation measures. If liquefaction were to occur, it could potentially cause some differential settlement of the embankment, which in turn could lead to structural distress of the pavement. Rebuilding damaged roadways is rapid and significantly more economical than attempting to mitigate the potential for liquefaction and is the preferred course of action at this time.

### 8.2 Cuts and Excavations

Based on our review of the preliminary project plan and profile drawings, cuts for the proposed improvements are generally on the order of less than 1 m in depth. Profile grade sheets are included in the Appendix to this report. These cuts will be temporary for the relatively minor (0.9 m) sub-excavation of the roadway. The proposed excavations are anticipated to expose loose, moist soils that have been disturbed by farming operations.

Excavation at the project site is expected to consist of cut and general grading. Excavation of the on-site materials may be performed utilizing conventional heavy-duty earthmoving equipment in good working order. No blasting operations are anticipated.

Temporary shallow excavations up to 1.5 m in depth should be generally stable, except in clean sandy material. In order to satisfy OSHA requirements, excavations that appear unstable, or are deeper than 1.5 m, and which will be entered by workers, should be shored or the side slopes should be laid back at a slope inclination of 1:1.5 (vertical:horizontal) or flatter.

Topsoil, vegetation, pavement, and other deleterious materials should be removed from the cut areas prior to the excavation operations. All excavated materials that are unsuitable for use as fill should be removed from the site. The total depths and limits of clearing should be performed in accordance with Section 16 of the Caltrans Standard Specifications. Cut slopes should not have a finished inclination of steeper than 1:2 (vertical:horizontal).

### 8.2.1 Grading Factors

The grading factor for this phase of the project is currently being determined by the District 11 Pavement Engineering Section. A grading factor of 0.92 had been established for the Phase 2 project. It is anticipated that the grading factor for this phase of the project will not be significantly different than for the previous phase.

## 8.3 Embankments

The SR-111 alignment between KP 28.2 and KP 32.5 is planned as a relatively minor embankment section along the entire project corridor. The anticipated fill height (not including structural section) will vary from approximately 0.6 to 1.5 m, and the pavement structural section height is about 1.2 m. Side slopes are anticipated to be 1:6 (vertical:horizontal) or flatter for inside median and outside slopes. Fill associated with the proposed improvements should be placed and compacted in accordance with Section 19 of the Standard

Specifications.

The upper 0.9 m of the existing soils are soft in consistency due to farming operations. In order to provide a suitable embankment foundation it is recommended to: 1) remove the upper 0.6 m of the existing soils within the roadbed prism and stockpile; 2) scarify, moisture condition and compact the next lower 0.3 m to 90 percent relative compaction, and 3) replace the removed 0.6 m as compacted fill in accordance with Section 19-5 of the Standard Specifications. Measured ground water levels along the project alignment ranged between 0.8 and 3.3 m below existing ground surface. In general, the existing ground water conditions should have no significant impact on the project. Along sections of the alignment where the ground water level may be less than 1.0 m as evidenced at Well Location P-1, special measures will be required to permit construction to proceed. In these areas, it is recommended that the existing soils be excavated to a depth of 0.9 m. The bottom of the excavated area should be stabilized using a combination of woven geotextile material and Class 2 Base Material. The placment of the geotextile and base material should be accomplished as follows:

- Place the geotextile at the bottom in the area covering the proposed width of the roadway
- Place and compact 0.6 m of Class 2 base material on the top of the geotextile in accordance with the standard specifications.
- Encapsulate the class 2 Base material by wrapping the geotextile around the Class 2 base material. The geoextile should cover the top of the Class 2 backfill with a minimum overlap of 0.6 m.
- Place and compact embankment fill over the encapsulated base material until pavement subgrade elevation is attained.

The geotextile should be a woven geotextile conforming to the following property values and specifications.

| Property | Test Method | Unit | Minimum Average Roll Value |
|---|---|---|---|
| Grab Tensile Strength | ASTM D4632 | kN | 1.34 |
| Grab Tensile Elongation | ASTM D4632 | % | 15 |
| Mullen Burst Strength | ASTM D3786 | kPa | 4134 |
| Puncture Tear Strength | ASTM D4833 | kN | 0.53 |
| Trapezoid Tear Strength | ASTM D4533 | kN | 0.51 |
| Permeability | ASTM D4491 | cm/sec | .001 |
| Flow Rate | ASTM D4491 | $l/min/m^2$ | 81 |

As an alternative to geotextile, other methods of in situ soil stabilization may be utilized in wet areas. A lime slurry mix may be used but this method of stabilization is best suited to clayey subgrade materials. If sandy, silty soils are exposed in the subgrade, the use of fly ash may prove to be more effective.

We recommend that surface vegetation or organic topsoil from all areas that will receive fill be stripped according to section 16 of the Standard Specifications.

Minor settlement of the embankment and subgrade soils on the order of 20 mm or less is expected following placement of minor fills and structural section. Overconsolidated clays underlie the proposed alignment. It is likely that these clays have been preconsolidated under the weight of ponded water during past episodes of flooding in the Imperial Valley. The overconsolidation ratio (OCR) was calculated based on consolidation tests performed on several samples of clay during the Phase 2 study. Values of OCR ranged from 3.1 to 7.1 with a typical value of 3.6. The majority of settlement is expected to occur during construction, thus a waiting period is not warranted.

## 9. Corrosion Studies

The results of corrosion testing and culvert recommendations are contained in the Materials Design Report for the project.

## 10. Material Sources

An assessment of possible off-site material sources will be presented in a separate Materials Information Brochure.

## 11. Construction Considerations

### 11.1 Differing Conditions

Differing site conditions are conditions that were not encountered during the site investigation, or are latent physical conditions that differ materially from those indicated within this Geotechnical Design Report. It is imperative that the designers, the resident engineer, and/or the contractor notify the geotechnical staff of roadway Geotechnical South immediately upon recognition of any condition differing from that described within this report. Roadway Geotechnical South can be reached through Joe Egan at (858) 467-4051.